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SEISMIC DEMAND ON SHORT PERIOD RC STRUCTURES UTILIZING A NEW MEASURE OF PULSE-TYPE GROUND MOTION SEVERITY

Mehrdad SASANI[1]

SUMMARY

Measures (parameters) of ground motion severity are required for estimating seismic hazard. Structures subjected to severe seismic ground motions undergo inelastic response and proper measures of ground motion are required to be well correlated with responses of interest. In this paper, it is shown first that under pulse type ground motions, inelastic response of short period structures (having periods less than about 1 sec) is significantly affected by acceleration pulses. Then, an equivalent rectangular acceleration pulse, called significant peak ground acceleration (*SPGA*) is defined, which is well correlated with inelastic response of short period structures.

It is shown that for short period structures, modeling a pulse-type ground motion by a simple pulse form that matches well with the main velocity pulses of the ground motion does not lead to a reliable prediction of the response of short period structures. This holds even for structures having periods about one half of the period of the simple pulse form. It is also shown that the *SPGA* correlates significantly better than other available measures with the inelastic response of short period structures.

Using main characteristics of structures, a relationship between the *SPGA* and the response of short period reinforced concrete (RC) structures is developed. It is shown that the relationship reliably predicts the inelastic response of short period structures to pulse type ground motions. The relationship can be used to determine the strength and deformation demand on such structures.

INTRODUCTION

Under severe seismic ground motions, most structures undergo deformations beyond their yield deformations. This is mainly due to the fact that designing structures to resist severe ground motions elastically is cost prohibitive. Seismic ground motions could be categorized as general and pulse type. Pulse-type ground motions that have been recorded near causative faults are characterized by significant velocity and displacement pulses (Somerville [1]). Examples of pulse-type ground motions are the recorded motions at the Rinaldi station during the 1994 Northridge, CA earthquake and at the Takatori station during the 1995 Kobe, Japan earthquake. In spite of the warnings by Bolt [2] on the existence of large ground velocity and displacement pulses (flings) in the recorded motions near the seismic sources, and the studies conducted by Bertero [3] on the significance of acceleration pulses on the inelastic

[1] Assistant Professor, Northeastern University, Boston, USA. Email: Sasani@neu.edu

response of structures, it was only after the 1994 Northridge, California and 1995 Kobe, Japan earthquakes that pulse-type ground motions have become the focus of several studies and were explicitly considered in building codes (Uniform Building Code [4]). Perhaps, mainly because of the long duration velocity and displacement pulses, studies have been more focused on the response of tall buildings to pulse-type ground motions. However, such ground motions can impose significant demands on short period structures, as well (Bertero [3], Sasani [5]). In the United States about 99.8% of office buildings are less than ten stories, which provide about 93.5% of the total floor space. These percentages are consistent for the West, South, Midwest, and Northeast parts of the United States (Department of Energy [6]). Inclusion of residential buildings is likely to increase the percentages. Therefore, the focus of this study is on the inelastic response of short period structures (T= 1.0 sec) to pulse-type ground motions.

In order to reliably design new and rehabilitate existing structures against severe pulse-type ground motions, there is a need to identify the main characteristics of such ground motions that control the nonlinear dynamic response of structures. In this paper a newly defined measure of the ground motion severity for pulse-type ground motions is improved and further examined, and an equation for estimating the nonlinear response of short period RC structural wall systems to such motions is developed.

Under seismic ground motions, the damage in a structure depends on the force-deformation histories that are developed in its members. In general, the damage depends on the strength and deformation capacities and demands of each member, as well as the input and dissipated energies. However, for near-source events, where the ground motion is dominated by a few pulses, the effect of cumulative energy dissipation is less significant. Thus, with focus on near-source ground motions, in this paper a probabilistic model is developed for the strength and deformation demands of RC structural walls, but the effect of dissipated energy is not considered.

EQUIVALENT SIMPLE PULSE FROMS

Because the main characteristic of near-source ground motions is that they contain distinct velocity and displacement pulses, some researchers have proposed simple velocity and displacement pulse forms that best match the ground motion pulses (Makris [7], Alavi [8]). However, because such simple pulse forms do not necessarily match the ground acceleration (Makris [7]) and because short period structures are more affected by the acceleration pulses rather than by the velocity pulses of the ground motion, such simplification may not be appropriate for short period structures.

The procedure proposed by Alavi [8] determines simple pulse forms to represent near-source ground motions. In the remaining of this paper, such pulse forms are called “the simple pulse forms”. Using three predefined shapes for the simple pulse forms, the procedure identifies main parameters of the predominant pulse in the near-source ground motion. These parameters are the pulse period (T_p), and the pulse severity (i.e. effective acceleration, a_{eff}). To determine the correct pulse shape the authors suggest using a judgmental approach when inspecting the time history of the ground motion and comparing the ground motion and simple pulse form spectral shapes (the velocity and displacement response spectra). The pulse period is one of the two parameters to be identified, which can be defined by a peak in the elastic velocity response spectrum. There are one simple and one complex way to estimate the pulse severity parameter, a_{eff}. In the simple procedure, $a_{eff} = 4\ PGV / T_p$, where *PGV* is the peak ground velocity of the motion. The authors have postulated that the equivalence between a near-source record and the corresponding simple pulse form can be reasonably established for $0.375 \leq T/T_p \leq 3.0$, where T is the period of structures.

Table 1 lists sixteen pulse-type ground motions recorded near causative faults that are used in this paper. The ground motions have minimum peak ground acceleration (*PGA*) and peak ground velocity (*PGV*) of

Table 1. Main characteristics of pulse-type ground motions

No.	Earthquake	Year	Station	Comp.	*PGA* (g)	*PGV* (m/s)
1	Tabas, Iran	1978	Tabas	TR	0.85	1.21
2	Imperial Val., CA	1979	Array 6	230	0.44	1.10
3	Morgan Hill, CA	1984	Coyote	285	1.30	0.81
4	North Palm Spring, CA	1986	North Palm	210	0.59	0.73
5	Superstition, CA	1987	Parachute	225	0.46	1.12
6	Loma Prieta, CA	1989	Los Gatos	000	0.56	0.94
7	Cape Mendocino, CA	1992	Cape Mend.	000	1.52	1.28
8	Cape Mendocino, CA	1992	Petrolia	090	0.66	0.90
9	Landers, CA	1992	Lucerne	260	0.68	1.37
10	Erzincan, Turkey	1992	Erzincan	000	0.52	0.84
11	Northridge, CA	1994	Jensen (Gen.)	292	0.99	0.59
12	Northridge, CA	1994	Newhall	000	0.59	0.97
13	Northridge, CA	1994	Rinaldi	228	0.89	1.74
14	Northridge, CA	1994	Sylmar (O.V.)	000	0.73	1.22
15	Kobe, Japan	1995	JMA	000	0.82	0.81
16	Kobe, Japan	1995	Takatori	000	0.79	1.74

0.44 g and 0.59 m/sec, respectively. In order to examine the accuracy of the above procedure, the response of short period elastic-perfectly plastic single degree of freedom (SDOF) systems having damping ration, ξ=0.05, to nine of these records that are used by Alavi [8] is computed. Table 2 shows the estimated a_{eff}, using the above-mentioned simple procedure (the complex procedure is examined in the next section). Using the pulse shape and period for the nine ground motions as suggested for developing the single pulse forms, Figure 1 shows the ratios of the maximum displacements of the inelastic SDOF systems under recorded ground motions to those under simple pulse forms for short period systems (T<1.0 sec). Note that all the SDOF systems used in Figure 1 have $0.4 \leq T/T_p \leq 1.0$ with increments of 0.1.

As it can be seen in Figure 1, the response obtained from the simple pulse forms significantly underestimates the response of short period structures and shows significant scatter. The main reason that the procedure underestimates the nonlinear response of short period structures is discussed in more details in the next section.

Table 2. Effective acceleration using procedure developed by Alavi [6]

Station	a_{eff} (1) (g)
Array 6	0.13
Coyote	0.41
Los Gatos	0.13
Erzincan	0.15
Rinaldi	0.71
Olive View	0.21
Newhall	0.31
JMA	0.37
Takatori	0.35

(1): $a_{eff} = 4\, PGV / T_p$

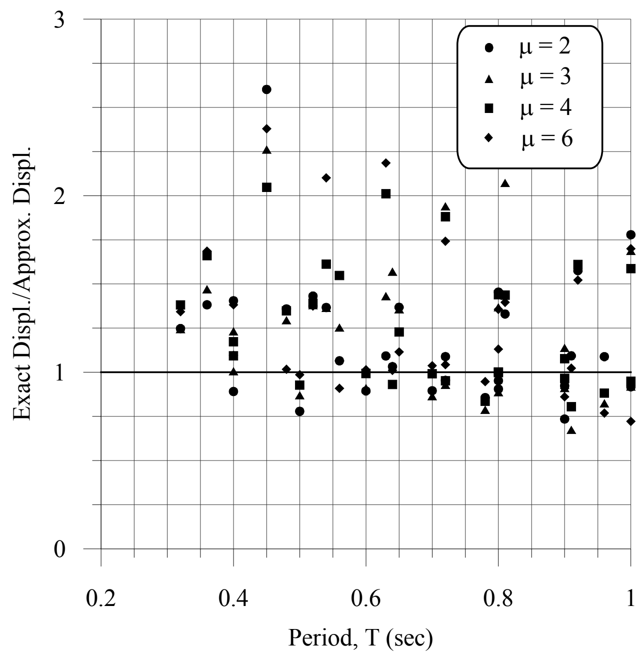

Figure 1. Displacement ratios for elastic-perfectly-plastic SDOF systems having 0.2sec≤ T ≤1.0sec and 0.4≤ T/T_p ≤1.0, using the simple pulse forms (ξ=0.05)

NONLINEAR RESPONSE OF SHORT PERIOD STRUCTURES TO PULSE-TYPE GROUND MOTIONS

The characteristic difference between critical ground motions for elastic and inelastic response of structures has been discussed by Bertero et al. [3]. They have pointed out that whereas harmonic type ground motions are critical for elastic response of structures having periods close to that of the ground motion, ground motions with severe acceleration pulses may be more critical for the response of inelastic structures. In order to study the behavior of yielding systems and effects of the relationship between the strength of the system and the severity of the ground acceleration on the response, consider an elastic-perfectly-plastic undamped SDOF system of yielding strength, $F_y = C_y m$. Note that C_y has the unit of g. The response of this system under a constant ground acceleration equal to C_y can be explained using the conservation of energy principle. The vertically hatched area in Figure 2 shows the input or external energy per unit mass and the gray area shows the sum of the dissipated and elastic energies per unit mass. In addition to the two internal energy terms, the mass of the system has kinetic energy. However, if the mass stops (at the maximum displacement) the kinetic energy is zero. At that time, the balance of the energy requires that the external and internal energies be equal, however, the input energy that is shown by vertical hatches is always larger than the internal energy. Therefore as long as the constant base acceleration continues, the displacement of the system increases. However, if the magnitude were somewhat smaller than C_y, the mass will stop and moves in the other direction. This example demonstrates the importance of the magnitude of the ground acceleration relative to C_y.

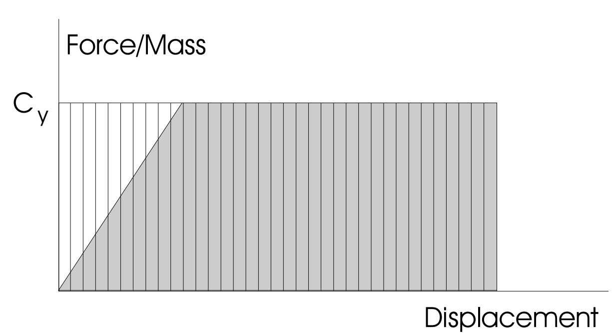

Figure 2. Force Displacement relationship and input energy

In the above example it was assumed that the constant ground acceleration was continuously applied. However, under recorded pulse-type ground motion, the acceleration pulses are applied over a limited time. Therefore, the reason the recorded ground acceleration pulses are more important for short period structures ($T \leq 1.0$ sec) is that the duration of the pulse will be considerable compared to the period of the structure. In order to illustrate the importance of an acceleration pulse in the response of short period nonlinear structures, the response of an elastic-perfectly plastic SDOF system having T=1.0 sec and a damping coefficient ξ=0.05 under the Takatori ground acceleration as well as under the corresponding simple pulse form are shown in Figure 3.

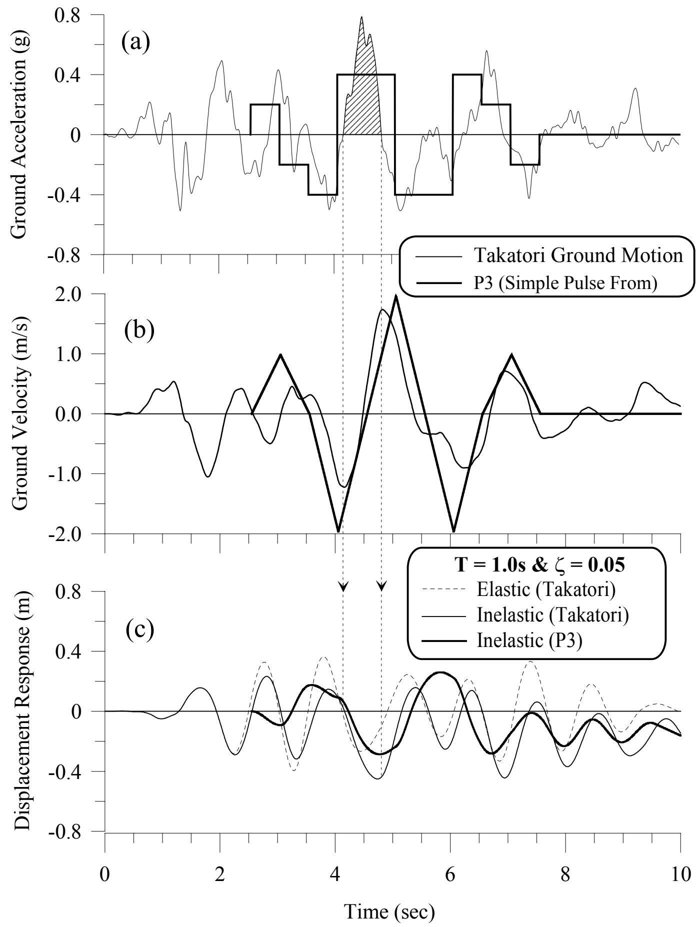

Figure 3. (a) & (b) Time histories of recorded ground motion of Takatori Station and corresponding simple pulse forms; and (c) Displacement response of elastic and inelastic SDOF systems

The yield strength is set such that the maximum displacement ductility (maximum displacement divided by the yield displacement) under the Takatori ground motion equals two. Figures 3(a) and (b) show the ground acceleration and velocity of the Takatori ground motion and those of the corresponding simple pulse forms. Note that for the simple pulse forms the a_{eff} is the value estimated from the complex procedure (Alavi [8]). It should also be noted that the simple velocity pulse form is shown over a duration of time that best matches the recorded ground velocity. Figure 3(c) shows the inelastic response of the SDOF system to both ground motions. Also shown is the elastic response of a SDOF system having the same period and damping ratio.

As can be seen in Figure 3(c), even though the simple velocity pulse form seems to be more severe than the recorded ground velocity, Figure 3(b), the response under the simple pulse form underestimates the maximum response of the system. This could be explained by the comparison of the recorded ground acceleration and that of the simple pulse form, Figure 3(a). As can be seen the simple acceleration pulse form significantly underestimates the recorded acceleration pulse, which is hatched. This is caused by the difference between the corresponding slopes of the ground velocities, see Figure 3(b). Inspecting the response of the inelastic SDOF system and relating the response to the corresponding acceleration pulse of the Takatori ground motion, it becomes clear that the hatched acceleration pulse has caused the maximum response of the system.

It is important to recognize that unlike the inelastic response, the elastic response (dashed curve) is maximized at some previous time. Comparing the elastic and inelastic responses to the Takatori ground motion shown in Figure 3(c) illustrates the characteristic difference between these two responses. As it can be seen, the maximum elastic displacement response occurs at about 3.25 sec where neither the main acceleration nor the main velocity pulses have started yet. Such a peak is mainly controlled by the amount of the input energy of the ground motion around the period of the system. However, the peak of the elastic-perfectly plastic system occurs right after the main acceleration pulse (hatched).

NEW MEASURE OF PULSE-TYPE GROUND MOTION SEVERITY

In the previous section it was shown that a severe ground acceleration pulse can significantly control the response of inelastic short period structures. Based on such an understanding, in this section a new measure of ground motion severity for pulse type motions is introduced and it is shown that compared to the simplified pulse forms discussed in the previous section, the new measure is considerably better correlated with inelastic response of short period structures. The new measure was first presented by Sasani [5] and [9] and here the measure is further improved. The new measure is called the significant peak ground acceleration (*SPGA*). The *SPGA* is defined as the maximum ratio of the significant variation of ground velocity (*SVGV*) and its duration. The *SVGV*, in turn, is defined as the maximum variation of the ground velocity over a time period.

Minimum Duration of Pulse

Because acceleration is the time derivative of velocity, if no minimum duration is considered over which the *SVGV* is found, the *SPGA* will be equal to the *PGA*. However, if the *PGA* of ground motions is associated with sparks (short duration pulses), unless for very short period structures (less than about 0.2 sec), the response of structures and in particular the inelastic response of structures is not well correlated with the *PGA*. In other words, as discussed before, acceleration pulses with considerable duration control the inelastic response of short period structures. Therefore, in order to have a ground motion measure that is well correlated with inelastic response of structures, a minimum duration needs to be defined in estimating the *SVGV* and the *SPGA*. Such a minimum duration is found below.

In order to define a minimum duration for acceleration pulses, we make use of the Bayesian parameter estimation technique. Details of the Bayesian technique can be found in the existing literature (Box [10] and Der Kiureghian [11]). For this purpose, given the relation between the acceleration pulses and the response of short period structures as described above, the following linear relationship between C_y, and *SPGA* is used:

$$\hat{C}_y = (\theta_1 + \theta_2 T + \theta_3 \mu)\, SPGA \qquad (1)$$

where θ_1, θ_2, and θ_3 are model parameters. The superposed hat on C_y indicates that the model is idealized and subject to error. In order to have a normally distributed error term (with zero mean, if the model is to be unbiased) linearly added to the model and to have a constant coefficient of variation (homoskedasticity assumption) for C_y, (1) needs to be properly transformed. Considering the non-negative nature of C_y, the error term is added after taking logarithm from both sides of (1), yielding

$$Ln(C_y) = Ln\left[\left(\theta_1 + \theta_2 T + \theta_3 \mu\right) SPGA\right] + \varepsilon_{cy} \quad (2)$$

In order to define the proper duration for *SPGA*, the standard deviation σ_{cy} of ε_{cy} is estimated for different values of the duration. To estimate σ_{cy} first the inelastic response C_y of elastic-perfectly plastic SDOF systems with periods 0.2s ≤ T ≤1.0s and displacement ductility ratios μ = 2, 3, 4, and 6 under the sixteen near-source ground motions listed in Table 1 are calculated. This data is then used in conjunction with the Bayesian updating rule to estimate the statistics of σ_{cy}. It is found that the minimum value of σ_{cy} equal to 0.27 occurs for a duration of 0.3 sec of the acceleration pulse. Therefore, the *SVGV*, is defined as the maximum variation of the ground velocity over a time period no less than 0.3 sec. Figure 4 demonstrates the definition of *SPGA*. The values of *SPGA* for the sixteen recorded ground motions are given in Table 3. ATC 3-06 [12] defines effective peak acceleration (*EPA*) and effective peak velocity (*EPV*). *EPA* is defined as the best fit to 5% damping elastic acceleration response spectrum for 0.1 ≤T≤ 0.5 sec. *EPV* is defined as the best fit to 5% damping elastic velocity response spectrum for T about 1 sec. For comparison with the *SPGA*, the values of *EPA* and *EPV* for the sixteen ground motions are also provided in Table 3.

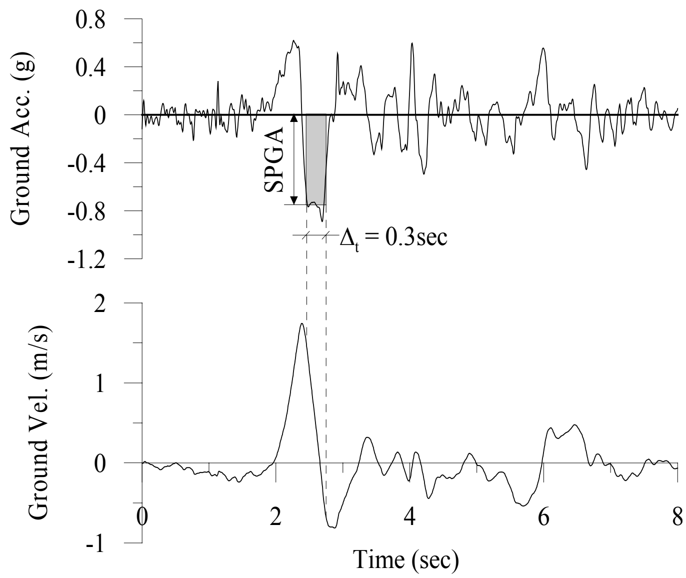

Figure 4. Graphical representation of *SPGA*

One commonly used measure of the ground motion severity for estimating the seismic response of inelastic structures is the elastic response spectrum. Figure 5 shows the relationship between the elastic acceleration response spectra S_a and the yield coefficient, C_y (inelastic acceleration spectra) for a SDOF system with T = 0.7 sec and a displacement ductility μ=4 for the sixteen ground motions. Also shown in the figure is the relationship between the *SPGA* and C_y. As can be seen, in spite of the fact that the *SPGA* is independent of the period of the structure, there is a much better correlation between the *SPGA* and C_y than between the S_a and C_y. While the correlation coefficient between the *SPGA* and C_y is about 0.91, the correlation coefficient between S_a and C_y is only 0.55. The mean of the correlation coefficient between the *SPGA* and C_y over a family of SDOF systems with 0.3sec = T = 0.9sec and 2 = μ = 8 is 0.82 with a minimum value of 0.7. For the same SDOF systems, the mean correlation coefficient between the S_a and C_y is 0.60 with a minimum value of 0.33. This implies that for short period structures, construction of inelastic response spectra from an elastic one is not appropriate.

Table 3. *SPGA*, *EPA*, and *EPV* for ground motions

No.	Station	*SPGA* (g)	*EPA(g)*	*EPV (m/s)*
1	Tabas	0.32	1.02	0.46
2	Array 6	0.24	0.28	0.28
3	Coyote	0.40	0.59	0.58
4	North Palm	0.25	0.46	0.50
5	Parachute	0.35	0.31	0.70
6	Los Gatos	0.33	0.52	0.77
7	Cape. Mend	0.35	0.90	0.45
8	Petrolia	0.33	0.46	0.59
9	Lucerne	0.25	0.45	0.28
10	Erzincan	0.36	0.29	0.38
11	Jensen (Gen)	0.27	0.77	0.66
12	Newhall	0.49	0.44	0.79
13	Rinaldi	0.75	0.59	0.97
14	Sylmar (O.V.)	0.37	0.54	0.57
15	JMA	0.53	0.49	0.79
16	Takatori	0.66	0.39	1.29

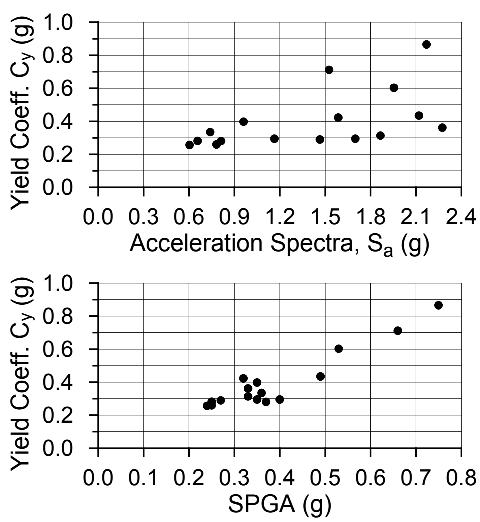

Figure 5. Correlation between S_a and C_y and between *SPGA* and C_y for SDOF systems with T =0.7 sec, μ=4, and ξ=0.05

Improved relationship for estimating C_y

In order to improve the estimation of C_y, using *SPGA*, the following more general relationship is examined.

$$Ln(C_y) = Ln\left[\left(\theta_1 + \theta_2 T^{\alpha} + \theta_3 \mu^{\beta}\right) SPGA\right] + \varepsilon_{cy} \tag{3}$$

Again using the Bayesian technique, it is found that σ_{cy} is minimized for α and β having mean values almost equal to 1 and –1, respectively. In order to reduce the number of the parameters of the model in (3), the mean of these two parameters are used as point estimators of the parameters α and β. Therefore, the following relationship is used to estimate C_y.

$$Ln(C_y) = Ln\left[\left(\alpha_1 + \alpha_2 T + \alpha_3 / \mu\right) SPGA\right] + \varepsilon_{cy} \tag{4}$$

Which can be rewritten as

$$C_y = e^{\varepsilon_{cy}} \left(\alpha_1 + \alpha_2 T + \alpha_3 / \mu\right) SPGA \tag{5}$$

It should be noted that under a given ground motion the required C_y for an inelastic SDOF system decreases as μ increases. Therefore, in (2) the mean value of θ_3 is found negative, while in (4) the mean value of α_3 is positive, which are consistent with the previous statement. The posterior estimated means, standard deviations, and correlation coefficients of the parameters in (4) and (5) are listed in Table 4. As it can be seen, the mean standard deviation of the error is only 0.24.

Table 4. Posterior statistics of yield coefficient model parameters

Parameter	Mean	Stan Dev.	Correlation Coefficient			
			α_1	α_2	α_3	σ_{cy}
α_1	1.05	0.072	1			
α_2	-1.05	0.073	-0.78	1		
α_3	2.45	0.167	-0.66	0.10	1	
σ_{cy}	0.24	0.012	0.00	0.00	0.00	1

Estimating displacement response

In estimating the maximum displacement of elastic-perfectly plastic SDOF systems having ξ=0.05, the following equation is used

$$\Delta_{\max} = \mu \frac{T^2 \ C_y}{4\pi^2} \tag{6}$$

where C_y is presented in (5) whose coefficients statistics are provided in Table 4. Therefore, given T, μ, and $SPGA$, one can estimate Δ_{max}, using (6).

Comparison of results

Figure 6 shows the ratios of exact maximum displacement of elastic-perfectly plastic SDOF systems under recorded ground motions (Table 2) and approximate maximum displacement, using mean estimation (5) and (6). As it can be seen, compared to Figure 1 the results in Figure 6 show considerably less scatter in the prediction of the maximum displacement. This is achieved in spite of the fact that the SDOF systems used in Figure 6 includes $T/T_p \leq 1.0$ with no lower bound on T/T_p, while in Figure 1, $T/T_p < 0.4$ are excluded, as suggested by Alavi [8].

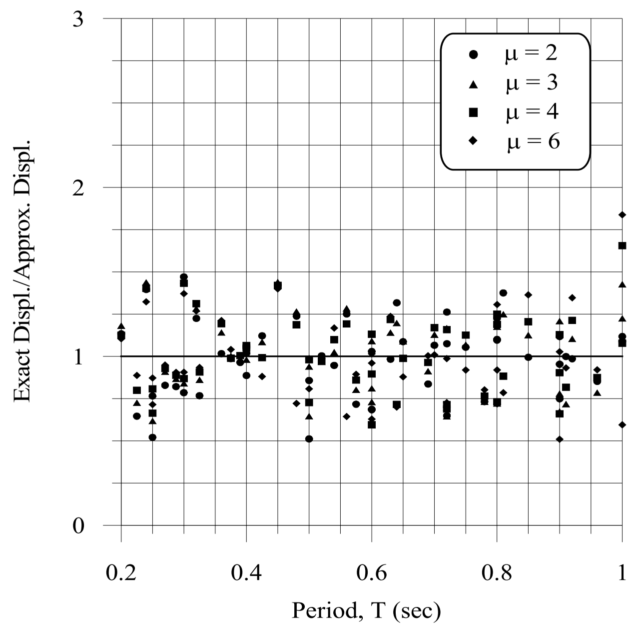

Figure 6. Displacement ratios for elastic-perfectly-plastic SDOF systems having 0.2sec≤ T ≤1.0 sec and T/T_p ≤1.0, using *SPGA* (ξ=0.05)

COMPARISON OF ESTIMATIONS

In the previous section, it was shown that the inelastic response of short period SDOF systems is significantly better correlated with the *SPGA* than with either simple pulse forms or elastic response spectra. In this section, the efficiency of some other measures of ground motion severity in predicting the inelastic response of SDOF systems is compared with the predictions based on the *SPGA*. The measures used are the *PGA*, *EPA, PGV*, and *EPV*. The values of *PGA* and *PGV* are given in Table 1, and the values of *EPA* and *EPV* are given in Table 3.

Using expressions similar to (4) but replacing *SPGA* with, *PGA*, *EPA*, *PGV*, and *EPV* the parameters of the models are estimated. The normalized relative error in predicting C_y using these measures against the period of elastic-perfectly-plastic SDOF systems are shown in Figure 7. In the figure, the mean, m, and the standard deviation, σ, are given for each case. As can be seen, compared to the models based on *PGA*, *EPA*, *PGV*, and *EPV*, the model based on the *SPGA* has significantly less scatter. Therefore, the *SPGA* is considered as a more reliable measure of the ground motion severity than other measures studied in this paper for predicting the inelastic response of short period structures under pulse-type ground motions. In other words, the *SPGA* is an efficient measure of the severity for pulse-type seismic ground motions.

RESPONSE OF STRUCTURAL WALLS

In this section the response of multi-degree-of-freedom (MDOF) systems under pulse type ground motions is examined. Again, the *SPGA* and other measures of ground motion severity are used to estimate the response of MDOF systems. 102 RC structural wall systems of four, six, and nine stories with natural periods between 0.3 and 0.9 sec are analyzed under the sixteen ground motions given in Table 1. The ratio of the lateral to vertical tributary areas for the walls varies between one and three. Lumped plasticity with bilinear moment-rotation relationships having three percent strain hardening are used for the wall sections. The wall section is kept constant over the height of each wall. The damping ratio in the first two modes is set equal to 0.05. The following relationship is used to estimate the required flexural strength at

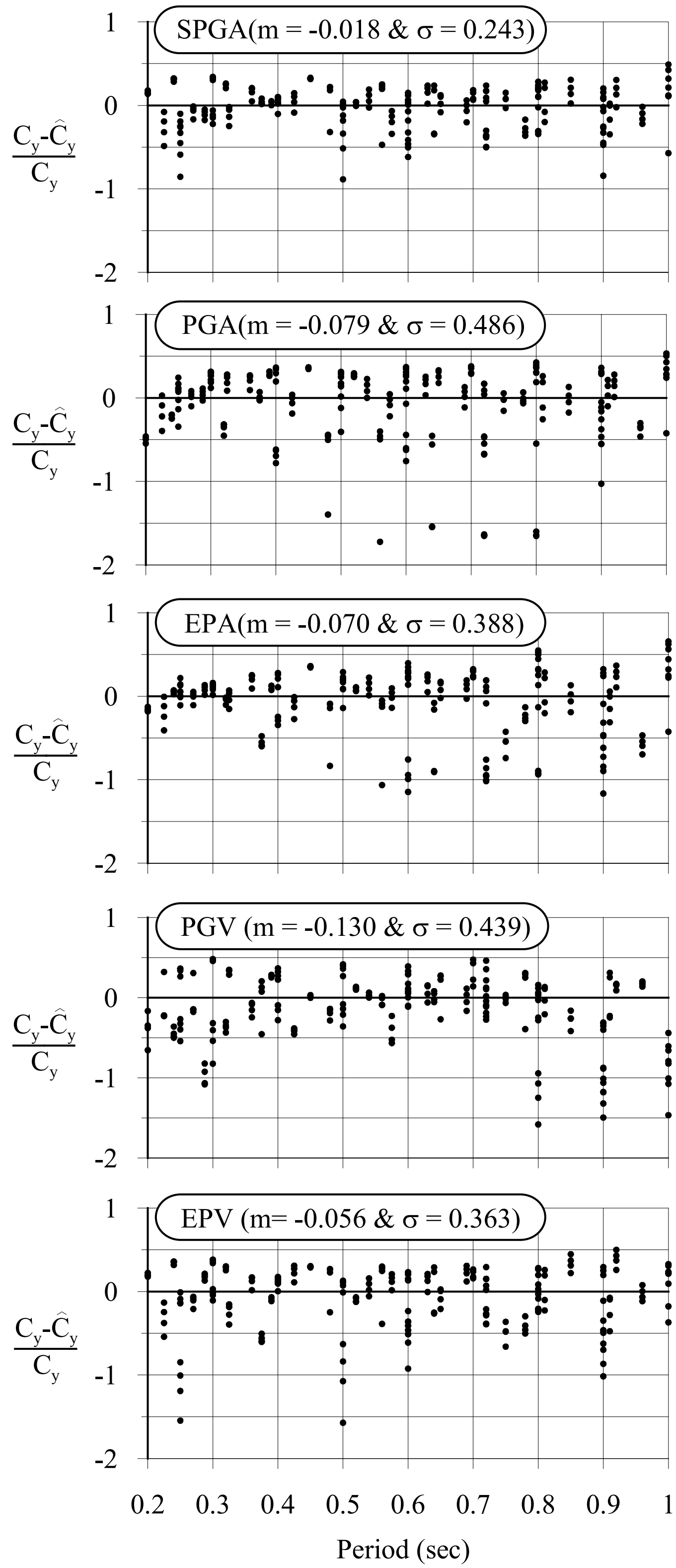

Figure 7. Comparison of relative error in estimating C_y using *SPGA*, *PGA*, *EPA*, *PGV*, & *EPV* for 0.2s ≤ *T*≤ 1.0s

the base of the structural walls

$$M_y = e^{\varepsilon_{My}} \left(\beta_1 + \beta_2 T + \beta_3 / \mu_\theta \right) H \; m_t \; SPGA \tag{7}$$

where M_y is the flexural yield strength, H is the total height, m_t is the total mass. μ_θ is the rotational ductility at the base equal to $1 + \theta_p / \theta_y$ where θ_p is the plastic rotation at the based and θ_y is the yield rotation (i.e. top yield deformation divided by H, under inverted triangular lateral loads). The posterior estimated means, standard deviations, and correlation coefficients of the parameters in (7) are listed in Table 5. As it can be seen, the mean standard deviation of the error is only 0.18.

Table 5. Posterior statistics of yield moment model parameters

Parameter	Mean	Stan Dev.	Correlation Coefficient			
			β_1	β_2	β_3	σ_{My}
β_1	0.58	0.042	1			
β_2	-0.42	0.033	-0.67	1		
β_3	0.87	0.068	-0.64	0.09	1	
σ_{My}	0.18	0.010	0.00	0.00	0.00	1

Using expressions similar to (7) but replacing *SPGA* with, *PGA*, *EPA*, *PGV*, *EPV* and S_a, the parameters of the models are estimated and the corresponding error in predicting M_y using these measures are 0.44, 0.48, 0.35, 0.28, and 0.34, respectively. That is, compared to the models based on *PGA*, *EPA*, *PGV*, and *EPV* and S_a, the model based on the *SPGA* estimates M_y with significantly reduced scatter. Therefore, the *SPGA* is considered as a more reliable measure of the ground motion severity than other measures studied in this paper for predicting the inelastic response of short period MDOF structures under pulse-type ground motions.

CONCLUSIONS

It is shown that the response of inelastic short period structures under a long duration acceleration pulse is significantly affected by the intensity of the pulse. Carrying out nonlinear dynamic analysis of elastic-perfectly plastic SDOF systems, it is concluded that the use of simple pulse forms that match main ground velocity pulses considerably underestimate the inelastic response of short period structures under severe pulse-type ground motions. Instead, it is shown that proper approximation of the main acceleration pulse, significantly improves the prediction of the response of short period structures.

A measure of the ground motion severity, *SPGA*, is presented that compared to other available measures, including *PGA, EPA. PGV, EPV*, and S_a, considerably better correlates with response of short period structures. The *SPGA* is defined as the maximum ratio of the significant variation of ground velocity (*SVGV*) and its duration. The *SVGV*, in turn, is defined as the maximum variation of the ground velocity over a time period no less than 0.3 sec.

Using the *SPGA*, an equation is presented that reliably predicts the response of inelastic short period SDOF systems. The coefficient of variation for such predicted response is found equal to 0.24. The new measure of ground motion severity and the equation for predicting response of inelastic short period structures can be used in estimating probability of failure of structures under pulse-type ground motions.

Using the *SPGA*, a relationship is developed for estimating the strength and ductility demand on short period RC structural walls. The coefficient of variation for such estimation is found equal to 0.18, which is significantly smaller than that for other measures examined in this paper.

Given the fact that in the United States about 99.8% of office buildings are less than ten stories, which provide about 93.5% of the total floor space, *SPGA* and the demand models can considerably improve seismic design and assessment of structures.

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